AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

CCX.

(Vol. IX.—November, 1880.)

WEB STRAINS IN SIMPLE TRUSSES WITH PARALLEL OR INCLINED BOOMS.

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READ AT THE TWELFTH ANNUAL CONVENTION, 25TH MAY 1880.

The great number of elaborate contributions which technical literature has in late years received upon the general subject of framed structures, makes the idea of throwing new light upon, or investing with greater simplicity the branch under consideration within the limits of a brief paper, appear at first glance idle and presumptuous.

The great number and universal distribution of the structures embraced in the classification to which this discussion is directed, and the fact that the great part of the handbooks and treatises on this subject give solutions of this class of frames based on a false assumption, and,

so far as I know, all fail in attaining the desired directness and simplicity in their treatment may, however, be plead in justification. Upon these handbooks the great body of practical engineers are obliged to rely, for the want of time and opportunity for investigation, though often called upon to design street and highway bridges of moderate spans.

This reliance results in strain sheets which are invariably inaccurate as to the web members, and never approximately correct in the case of bowstring girders.

The ordinary method of determining the maximum effect of the moving load upon the web members of simple as well as of compound trusses, assumes that as the moving load passes over them each panel is fully loaded before the adjacent triangle in advance bears any part of the load.

In trusses with a single system of triangulation or those in which the web strains of any panel pass to the abutment through the web members of the adjacent panel, this assumption is obviously erroneous, for the instant the head of the load passes a panel joint of such a truss a part of it is transmitted by the floor system to the adjacent triangle of the same system.

With this fact in view, we may proceed to investigate the maximum vertical stresses produced by uniform moving loads in trusses of this class.

The simplicity of mathematical formulæ and their ready adaptability to practical use, depend in great measure upon the nomenclature through which they are deduced.

In investigating trusses divided into panels of equal length, the panel length is the most simple and natural unit of length.

The strains change only at panel points, we have to deal with panel loads, and if in designing the truss the rise or height and the panel length are made to have a simple numerical ratio, the adoption of the panel length as the linear unit will be found to greatly abridge the arithmetical computations.

In Fig, 1, Plate XXIV., D, E, represents a simple truss, having N equal panels. Then the panel length being the linear unit, $N=$ length of truss.

Let $w=$ the dead load per panel, and $w'=$ the moving load per panel. Let n represent the number and consequent distance of any panel point from the loaded abutment and $h=$ the height of truss.

The bridge has a single system of triangles, and its floor joists are not continuous over the panel points, conditions which subsist in nearly all highway and street bridges of moderate span.

Suppose the moving load to proceed from and cover the part of the truss from D to a point f between the panel points n and $n+1$, measured from abutment D, and represent the portion ef of the panel beyond n, covered by the load by y.

It is required to find the maximum shearing strain at any panel point n due to the uniform moving load w' per panel.

The moment the front of the moving load passes the panel point n, a definite portion is transmitted by the floor joists to the panel point $n+1$, in advance of the load.

The principle that the shearing stress due to a partial load at any point in a beam, is equal to the reaction at the unloaded abutment due to that load, less the portion of the load between the point and that abutment, makes it evident that the shearing stress at n borne by the diagonal n, will increase until the head of the moving load passes the point n, and that it will diminish before the load reaches the point $n+1$.

This principle may be expressed analytically as follows. Let $R=$ the reaction at E from the portion of the moving load between D and n, R' the reaction at that abutment from the portion on y that is between n and f, and let P represent the portion of the load on y, transmitted by the floor system to the panel point $n+1$. Then the shearing stress S_n at $n = R + R' - P$.

The reaction $R = \frac{Wx}{N}$ in which $W=$ the whole load borne by the truss between D and $n = (n - \frac{1}{2})\,w'$ and $x =$ the distance of the centre of gravity of W from D, or the sum of the products of each panel weight by its distance from D, divided by the whole weight $\left(\frac{2n-1}{2}\right) w'$

$$\therefore x = \frac{w' + 2w' + 3w' \ldots + (n-2)\,w' + (n-1)\,w' + \frac{n\,w'}{2}}{\frac{(2n-1)\,w'}{2}}$$

$$= \frac{\frac{n^2\,w'}{2}}{\frac{(2n-1)\,w'}{2}} \qquad \text{but } R = \frac{Wx}{N} = \left(\frac{2n-1}{2N}\right) w' \; \frac{\left(\frac{n^2\,w'}{2}\right)}{\left(\frac{2n-1.}{2}\right) w'} = \frac{n^2\,w'}{2N.}$$

and R' the reaction at E from the load covering the portion y of the panel n to $(n+1)$, since the distance of its centre of gravity from D is $n+\frac{y}{2}$ and its weight $w'\,y$, is $R'=\frac{\left(n\,y+\frac{y^2}{2}\right)}{N}w'$, and P the portion of the moving load on y, which is borne by the panel point $(n+1)$ is $w'\,y\times\frac{y}{2}$ or $P=\frac{w'\,y^2}{2}$.

The reaction at E due to the whole moving load in the position assumed, is therefore $R+R'=\frac{n^2\,w'}{2\,N}+\frac{ny+\frac{y^2}{2}}{N}$, and the weight between the point n, where the shearing stress is sought and the unloaded abutment E is $P\times\frac{w'\,y^2}{2}$. Therefore the shearing stress at n is,

$$S_n=\frac{w'\,n^2}{2\,N}+\frac{ny+\frac{y^2}{2}}{N.}-\frac{w'\,y^2}{2}$$

$$\text{or } S_n=(n^2+2\,ny+y^2-N\,y^2)\frac{w'}{2\,N} \qquad (1).$$

This expression is obviously true for any value of y less than the panel length, and the value of y which renders it a maximum determines the position of the moving load which produces the greatest shear at the point n. We have by differentiation (neglecting the factor $\frac{w'}{2\,N}$ which is independent of y),

$$\frac{ds}{dy}=2\,n+2\,y-2\,Ny.$$

Making $$\frac{d\,s}{dy}=0. \quad y=\frac{n}{N-1.}$$

As the second dif. co-efficient is negative, this value of y renders the above general expression for the shearing strain a maximum.

Substituting this value of y in equation (1) and reducing we find

$$S_n=\frac{n^2\,w'}{2\,(N-1)} \qquad (2).$$

This is the general expression for the maximum shearing stress at any point n of a simple truss due to a uniform moving load w' per panel.

These vertical shearing stresses in framed beams with parallel chords are evidently wholly transmitted by the inclined braces a, b, c, etc., the induced strain being measured by the vertical shear at their end nearest

the loaded abutment, multiplied by the secant of their inclination to the vertical.

The whole stress in these braces, however, depends upon the combined action of the dead and moving load. The former since it is essentially constant and uniform in its action is correctly represented by the usual assumptions and accurately measured by the formulæ in common use.

By the nomenclature we have adopted, we can readily deduce the shear at any point of a simple truss due to the combined effect of the dead and moving load, and the resultant stress in any web member from their combined effect.

The shearing stress S' due to the dead load w with reference to the abutment E, numbering the panel points as before from D, at any point n, is $S'_n = \frac{(N-1)}{2} w - [N-(n+1.)]\ w = \frac{2n-(N-1)}{2} w.$

The greatest stress in any brace transmitting the moving load from n toward the unloaded abutment is therefore

$$(S_n + S'_n)\ sec.\ i = \left(\frac{n^2 w'}{2(N-1)} + \frac{2n-(N-1)\ w}{2}\right) sec.\ i.$$

If the brace be vertical, $sec.\ i = o.$

The solutions of this equation for values of n less than $\frac{N}{2}$ give the counter strains, and for values of n from $\frac{N}{2}$ to $(N-1)$ give the strains on the main ties and braces.

If the chords, as in the bowstring girder are not parallel, the stress in the web is increased or diminished thereby according as they diverge or converge from n toward E, though the principle determining the maximum shearing stress and its amount are not effected. The method of sections offers in this case the simplest analytical solution of the web strains.

If the bowstring $D\ E$ (Fig. 1) be cut by a vertical plane at any point as between n and $n+1$, the theory of equilibrium requires that the horizontal component of the stress in the compression member shall equal the sum of the horizontal components of the stresses in the tensile members.

The section of the arch A is obviously the compression member, and the section of the lower chord B and the diagonal C the tensile members.

If we represent the height of the arch at n by h_n and at $n+1$ by h_{n+1}, by the principle of the lever we have for the horizontal com-

ponent of the stress in the compression member A, due to the part of the moving load producing the maximum stress in the diagonal, $C =$ the maximum stress $\frac{n^2 w'}{2(N-1)}$ multiplied by the distance of the point n, from the unloaded abutment E, $= N - n$ divided by h_n, or $= \frac{n^2 w'}{2(N-1)} \times \frac{N-n}{h_n}$ and the horizontal stress in the section of the lower chord B $= \frac{n^2 w'}{2(N-1)} \times \frac{N-(n+1)}{h_{n+1}}$.

The horizontal component of the maximum stress in the diagonal C (or n) is, therefore, the difference of these horizontal stresses in A and B, or $= \frac{n^2 w'}{2(N-1)} \times \left(\frac{N-n}{h_n} - \frac{N-(n+1)}{h_{n+1}}\right)$ (3.)

The value of this expression mutiplied by $\frac{\text{the length of the diagonal}}{\text{the panel length}}$ gives the greatest stress on the diagonal n, due to the moving load.

If the truss be so designed that the panel points of one or both chords lie in the curve of equilibrium for an unbraced arch, the uniform dead load produces no stress in the diagonals.

For this case the above formula (3) can be simplified by deducing the values of h_n and h_{n+1} in terms of the known rise at the centre which we will represent by h.

The condition of equilibrium for the uniformly loaded, unbraced arch requires that the horizontal components of the stresses in all the arch members shall be equal. But the horizontal stress in any member of an arched truss is by the principle of the lever equal to the moment of flexure at its upper extremity divided by the vertical distance from that extremity to the lower chord. Therefore, the heights of the panel points of such a truss must vary as the moments of flexure at these points.

By the nomenclature we have adopted the moment of flexure at any point n, from a uniform load w per panel, is $m_n = \frac{wn}{2}(N-n)$.

The moment at the centre where the height is h, is $\frac{w N^2}{8}$

From the condition of equilibrium we have

$$\frac{w N^2}{8} : \frac{w n}{2}(N-n) :: h : h_n.$$

Therefore $h_n = \dfrac{4\,n\,h\,(N-n)}{N^2}$ (4.)

Similarly, $h_{n+1} = 4\,h\,\dfrac{(n+1)\,[N-(n+1)]}{N^2}$ (5.)

Substituting these values of h_n and h_n+_1 in equation (3), and reducing, we have for the horizontal component $H n$ of the maximum stress in any diagonal n of such a truss

$$H n = \frac{n^2 w'}{2\,(N-1)} \left(\frac{N^2}{4\,h\,(n^2+n)}\right) \qquad (6.)$$

Representing the vertical inclination of this diagonal by i, its maximum stress by S_n and the vertical component of this stress by V_n; since the horizontal component is to the vertical component as the panel length (unity) is to the vertical height of the diagonal n, or

$$V_n : \frac{n^2 w'}{2\,(N-1)} \left(\frac{N^2}{4\,h\,(n^2+n)}\right) :: \frac{4\,h\,(n+1)\,[N-(n+1)]}{N^2} : 1$$

we have $V_n = \dfrac{n\,w'}{2\,(N-1)}\,(N-n+1)$,

and $S_n = \dfrac{n\,w'}{2\,(N-1)} \times \Big(N-(n+1)\Big)\,\sec.\,i$ (7.)

The greatest thrust in any vertical web member in a bowstring of this class occurs when the diagonal between it and the head of the load is under its greatest stress, and equals the vertical component of the diagonal transmitting the load from its lower extremity towards the unloaded abutment, because, when the greatest strain occurs at any panel point n, the vertical at n is under tension from the part of the load transmitted to the loaded abutment, and the vertical at the panel point $n+1$, resists as a strut the whole stress then borne by the diagonal $n+1$.

Applying the principles and method employed above in deducing the stress in the diagonal at n, the horizontal component of the stress in the diagonal $n+1$ from the same shearing force $\dfrac{n^2 w'}{2\,(N-1)}$ is found to be $\dfrac{n^2 w'}{2\,(N-1)} \left(\dfrac{N^2}{4\,h\,(n+2)\,(n+1)}\right)$ and its vertical component or the greatest thrust produced by the moving load in the vertical $n+1$ is found by multiplying this expression by the ratio of the vertical height

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NOTE—Equation (4) affords the readiest means of determining in the process of design the exact heights of the several panel points of the arch from the axis of the lower chord.

of the diagonal $n+1$, to the panel length, and is

$$=\frac{n^2 w}{2(N-1)} \times \frac{N-(n+2)}{n+1} \qquad (8.)$$

As the verticals bear a tensile strain equal to the uniform dead load w per panel, less the weight of the arch, if we represent by w'' the weight of the arch per panel, the total strain in the vertical n is,

$$H=\frac{n^2 w'}{2(N-1)}\left(\frac{N-(n+2)}{n+1}\right)-(w-w'') \qquad (9.)$$

The maximum shears having been determined by equation (3) the graphical method furnishes a more rapid and direct means for determining the resulting stresses in the web members than the analytical formulæ we have deduced.

These shears represent that part of the entire reactions at the unloaded abutment producing the maximum stresses sought in the web members respectively, and may therefore be readily laid off to scale and traced through reciprocal figures to those web members to which they respectively apply.

For example, a strain sheet is required for a wrought-iron bowstring girder of 90 feet span, having a clear roadway of 18 feet and two sidewalks 6 feet wide, and having a rise at the centre of 10 feet.

Let the fixed points of the arch be in the curve of equilibrium and let it be divided into nine panels, and be required to carry a uniform moving load of 80 pounds per square foot of floor surface. The uniform dead load of such a truss, properly designed, will be about 400 pounds per lineal foot, and of its arch about 75 pounds per lineal foot. We, therefore have for the numerical values of our literal factors: $N=9$. $h=1$. $w=4\,000$ lbs. $w'=12\,000$ lbs. and $w''=750$ lbs.

To draw an accurate diagram of the frame, it is first necessary to determine the vertical heights of the panel points h_1, . $h_2 \ldots\ldots h_{(N-1)}$. This is readily done by substituting the numerical values for this case in equation (4.) and we have

$$h_1=\frac{4}{81.} \;(8) \times 10. = 3.951 \text{ feet.}$$

$$h_2=\frac{4\times 2.}{81.} \;(7) \times 10 = 6.914 \quad \text{"}$$

$$h_3=\frac{4\times 3}{81} \;(6) \times 10 = 8.889 \quad \text{"}$$

Fig 1.

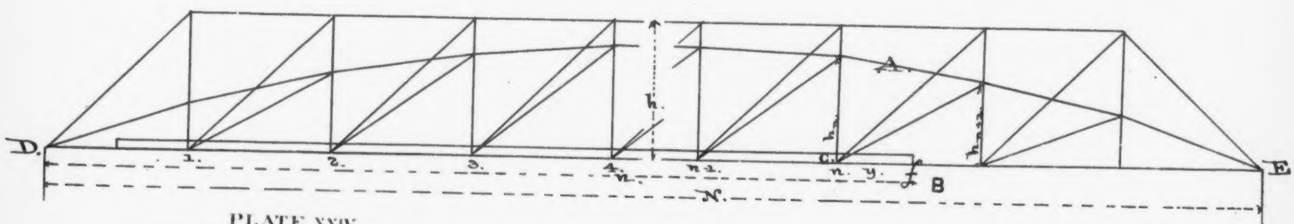


PLATE XXIV
TRANS. AM. SOC. CIV. ENG'R'S.
VOL. IX No. CCX
SWEET ON STRAINS

$$h_4 = \frac{4 \times 4}{81} (5) \times 10 = 9.876 \text{ feet.}$$

$$h_5 = \frac{4 \times 5}{81} (4) \times 10 = 9.876 \quad \text{"}$$

$$h_6 = \frac{4 \times 6}{81} (3) \times 10 = 8.889 \quad \text{"}$$

$$h_7 = \frac{4 \times 7}{81} (2) \times 10 = 6.914 \quad \text{"}$$

$$h_8 = \frac{4 \times 8}{81} (1) \times 10 = 3.951 \quad \text{"}$$

Draw the diagram of the frame D, E, Fig. 2, Plate XXV., to a convenient scale, with the diagonals required to transmit the panel weight towards the abutment E.

Draw to a convenient scale, say 10 000 pounds per inch, the force polygon A, B, O, for the frame fully loaded. That is, draw the vertical line $A\ B$, representing to the assumed scale 10 000 pounds per inch, the whole dead and moving load borne by the arch $(w + w') \times (N-1)$, and subdivide it into eight equal parts (the number of loaded panel points). Through the extremities and points of subdivision of $A\ B$, draw lines to the left, parallel to the several members of the arch. These lines will intersect at a common point O, and will represent, in direction and intensity, the stresses produced by the full dead and moving loads in the members of the arch to which they are respectively parallel.

The horizontal line $O\ F$, represents to the assumed scale of weight the stress throughout the lower chord, as well as in the horizontal member of the arch.

Through the vertex O, draw an indefinite vertical line, and lay off upon it, on a scale of, say 1 000 pounds per inch, from O, downward $O\ C$, the reaction at E, and from O, upward $O\ C'$, the reaction at D, produced at those points when the moving load exerts the greatest shearing force at the first panel, that is when n, in the formula $\frac{n^2\ w'}{(2\ N-1)}$ is unity.

On the principle of the lever, these reactions are in the proportion of 1 to 8. $O\ C, = \frac{(1)^2 \times 12\ 000}{16} = 750$ lbs. and $O\ C' = 6\ 000$ lbs. Through C, and C', draw horizontal lines intersecting $O\ A$, and $O\ B$, in G, and G'. $C\ G$, represents the stress in the first panel of the lower

chord and $O\ G$, the stress in the first panel of the arch from E, due to the reaction $O\ C$. Produce $C\ G$, representing the lower chord, and draw the lines a 7, b 6, c 5, d 4, e 3, f 2, g 1, parallel to the web members, taken in their order from E, and terminating at their lower ends in the lower chord produced, and at the other end in the lines of the polygon representing the panel sections of the arch to which they respectively belong.

These lines represent in direction and in intensity (to the assumed scale of 1 000 pounds per inch) the stress produced in all the web members by the reaction $O\ C$, (750 pounds), and the maximum stress in numbers 9 and 2 due to the moving load. The accuracy of the work is tested by drawing upward from the upper extremity of g the vertical 1, which should intersect its panel member, $D\ A$, at G, and close the polygon of the reactions $O\ C$, and $O\ C'$, due to the whole moving load in the position we have assumed.

It will be seen that the general expression for the maximum shear at any point is composed of the constant factor $\frac{w'}{2\,(N-1)}$ multiplied by the square of the distance or number of the given panel point from the loaded abutment.

We can, therefore, derive the greatest stresses in all the web-members from the reciprocal figure just drawn from the maximum shear when $n = 1$ by multiplying the stresses represented by its several lines to the assumed scale by the squares of the distances of the respective diagonals from D: *i. e.*, multiplying f and 3 by 4; e and 4 by 9; d and 5 by 16; c and 6 by 25., &c. The scale of the members beyond the middle of the arch is so small, however, that it is better to derive them from the reaction of the greatest shear at or near the middle, as for

$$n = 5, \text{ where the shear } \frac{n^2\,w'}{2\,(N - 1.)} = 750 \times 25 = 18750.$$

Lay off this reaction $O\ C''$ on a scale of 5 000 lbs. to an inch, draw $C''\ G''$ parallel to the lower chord, and continue the reciprocal figure to the member of the arch next to the centre, *i. e.*, to c.

The lines C and 6 represent the direction and intensity of the greatest stress on the 5th diagonal and 6th vertical respectively, while the lines a, b and 7 bear the same ratio to the maximum stresses in the members they represent as the maximum shears at the panel points 6 and 7, bear

to that at point No. 5, and their greatest stresses are determined by multiplying b and 7 by $\frac{36}{25}$ and a by $\frac{49}{25}$.

It must be borne in mind that from the thrust in the verticals determined as above, the tension $(w - w'')$, produced by the uniform dead load is to be subtracted from each, and that when the moving load passes over the frame in an opposite direction a set of diagonals is required running in the opposite direction to those shown, which will evidently be similarly strained, and that the verticals will then be differently strained, depending upon their distance from E. No. 7 becomes No 2, No. 6 No. 3, and No. 5 becomes No. 4. The greatest of these vertical strains is of course to be provided for.

There remains the case of drawbridges of moderate spans with which the general practitioner has not infrequently to deal, and about which he is often mislead.

As these bridges are usually constructed, the assumption that they consist of three unequal spans, as AD, DD', $D'B$ in Fig. 3, Plate XXVI., is erroneous, since the panel points between D and D' are commonly rigidly fixed to the turntable, and the loads upon them are carried directly by the latter.

So constructed, the cords of the truss between D and D' may be considered horizontal in whatever manner the end spans may be loaded.

Suppose each of the end spans to be divided into N equal panels, and assume the panel length as the linear unit. Suppose the span $A\ D$ covered with the maximum moving load, and let w' represent the portion borne at each panel.

Take A as the origin of rectangular co-ordinates, and represent vertical and horizontal distances from these axes by y and n respectively.

With the beam thus loaded, the external forces acting upon it between A and D are the several panel weights acting vertically downward, and the reaction R of the abutment A acting in the opposite direction. The panel weights are known, but the reaction R is yet to be ascertained.

Now, by the principle of the lever, the bending moment at any panel point, distant n from A, is the sum of the moments with regard to the panel n of all the weights between A and n, or

$$\left\{ \overline{1 + 2 + 3 + \ldots (n-2) + (n-1)} \right\} w = n(n-1)\frac{w'}{2}$$

But the reaction of the abutment A produces a moment of the opposite kind $= - R\,n$. Therefore, the bending moment of this truss at any panel n is $m = n\,(n-1)\,\frac{w}{2} - R\,n$.

From the differential equation of the elastic curve, we have for any homogeneous beam $m = \frac{d^2 y}{dn^2}\,E\,I$. Therefore

$$E\,I\,\frac{d^2 y}{dn^2} = \frac{w'}{2}\,(n^2 - n) - R\,n.$$

Integrating once we have

$$E\,I\,\frac{dy}{dn} = \frac{w'}{2}\left(\frac{n^3}{3} - \frac{n^2}{2} - R\,\frac{n^2}{2} + C.\right)$$

Since the beam is horizontal at D, at that point $\frac{dy}{dn} = 0$ and $n = N$. therefore

$$\frac{w'}{2}\left(\frac{N^3}{3} - \frac{N^2}{2} - R\,\frac{N^2}{2} + C\right) = 0$$

and

$$C = \frac{R\,N^2}{2} + \frac{w'}{2}\left(\frac{N^2}{2} - \frac{N^3}{3}\right)$$

$$E\,I\,\frac{dy}{dn} = \frac{w'}{2}\left(\frac{n^3}{3} - \frac{n^2}{2}\right) - \frac{R\,n^2}{2} + \frac{R\,N^2}{2} + \frac{w}{2}\left(\frac{N^2}{2} - \frac{N^3}{3.}\right)$$

Integrating again, we have

$$E\,I\,y = \frac{w}{2}\left(\frac{n^4}{12} - \frac{n^3}{6}\right) - \frac{R\,n^3}{6} + \frac{R\,N^2 n}{2} + \frac{w}{2}\left(\frac{N^2 n}{2} - \frac{N n^3}{3}\right) + C.$$

For $y = o$; $n = o$. Since there is no deflection at A; $C = 0$.

For $n = N$ also $y = O$. Since there is no deflection at D.

Therefore

$$\frac{w'}{2}\left(\frac{N^4}{12} - \frac{N^3}{6}\right) - \frac{R\,N^3}{6} + \frac{R\,N^3}{2} + \frac{w'}{2}\left(\frac{N^3}{2} - \frac{N^4}{3}\right) = 0.$$

$$\frac{R\,N^3}{2} - \frac{R\,N^3}{6} = \frac{w}{2}\left(\frac{N^4}{3} - \frac{N^4}{12} + \frac{N^3}{6} - \frac{N^3}{2}\right)$$

$$\frac{6\,R}{12} - \frac{2\,R}{12} = \frac{w}{2}\left(\frac{4\,N}{12} - \frac{N}{12} + \frac{2}{12} - \frac{6}{12}\right)$$

$$8\,R = w'\,(3\,N - 4).$$

$$R = w'\left(\frac{3\,N - 4}{8}\right)$$

Substituting this value of R in the equation for the bending moment, we have $M_n = \frac{w'n}{2}\,(n-1) - \frac{w'n}{8}\,(3\,N - 4)$.

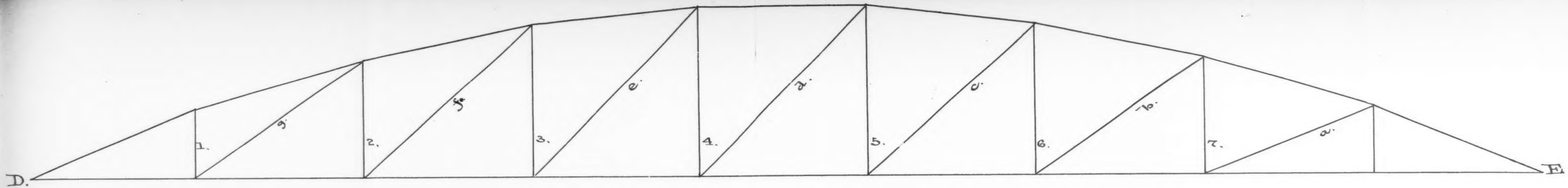


PLATE XXV
TRANS. AM. SOC. CIV. ENG'RS.
VOL. IX No CCX
SWEET ON STRAINS

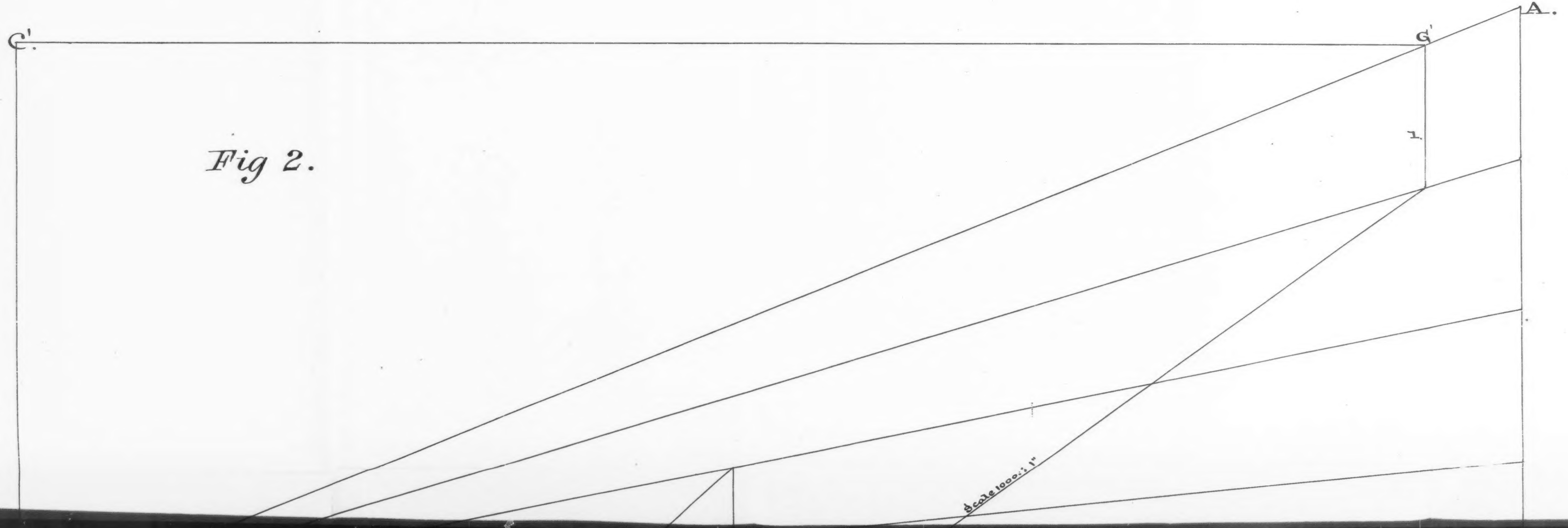
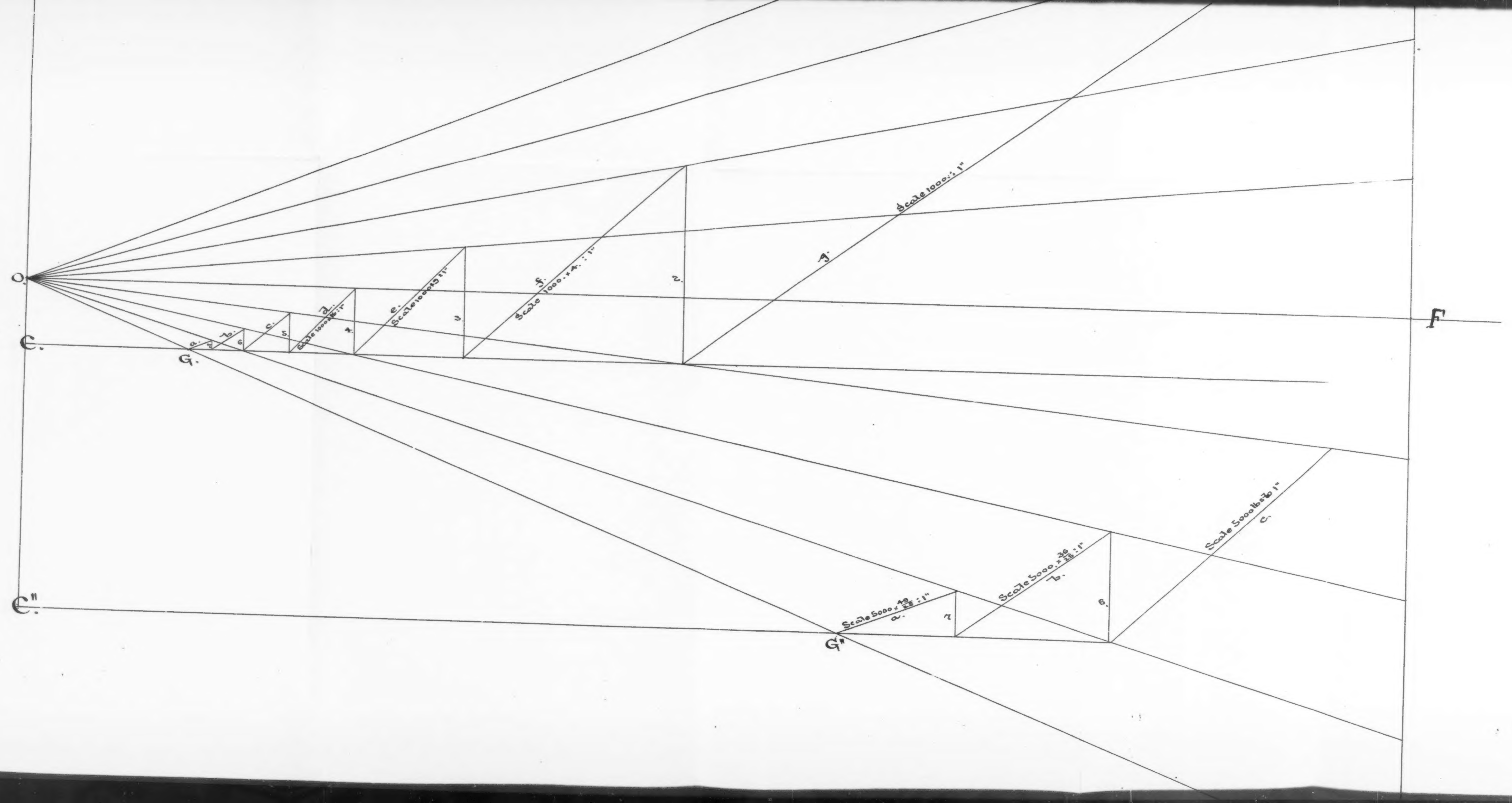
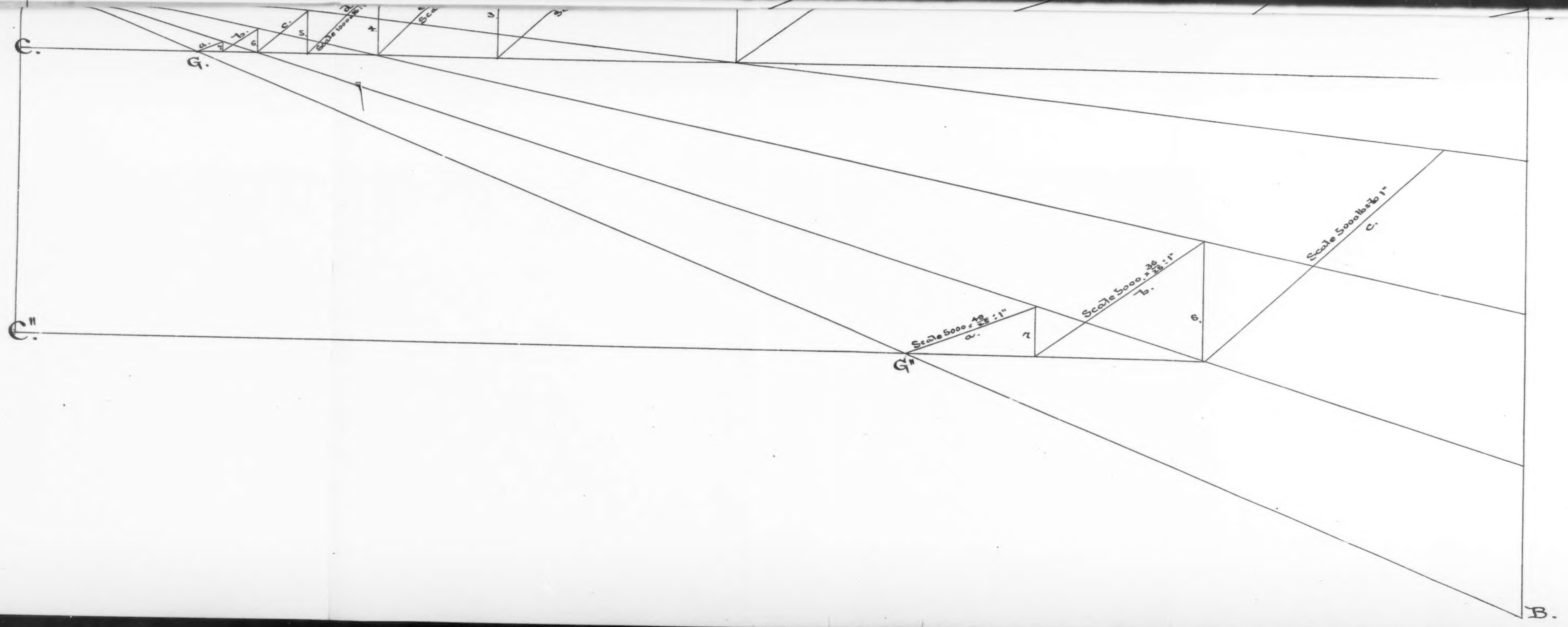


Fig 2.



O.
C.
G.
C''.
G''
F
a.
b.
c.
d.
e.
f.
g.
Scale 1000 x 3 : 1"
Scale 1000 x 4 : 1"
Scale 1000 : 1"
Scale 5000 x 49/25 : 1"
Scale 5000 x 36/25 : 1"
Scale 5000 lbs to 1"



C.
G.
a.
b.
c.
7
6
5.
4
3.
C"
G"
Scale 5000 × 49/25 : 1"
a.
7
Scale 5000 × 36/25 : 1"
b.
6.
Scale 5000 lbs to 1"
c.
B.

The bending moment at any panel point n determined by this formula when divided by the height of the truss in terms of the linear unit, gives the strain along the chord opposite the point n.

The web strains in the end spans are obtained by beginning at A or B, with the reaction R, which evidently measures the shearing force at the ends of the truss, and subtracting from this reaction one panel weight successively for the shearing strain at each successive panel point, and disregarding the algebraic sign of the result. These remainders must, of course, be multiplied by the ratio of the length of each brace to its vertical height, to obtain the stresses in the braces themselves. Let T be the strain in any pair of braces intersecting between panel point n and $n+1$. $T = (R - n\,w') \times \sec i$, in which $i =$ inclination of brace to vertical.

The greatest vertical strains in the braces of the panels over the turntable is the vertical component of the difference of the bending moments at D and D', when one end span is loaded, and the other bears only the dead load, while the maximum strains in the chords between D and D' are the same as at the panel D, or D' with the bridge fully loaded, as determined by the formula.

These formulæ give the maximum strains in any drawbridge of the kind above described due to the moving load, with either or both end spans fully loaded, in which, when the span $A\ D$ is loaded, and $D\ D'$ and $D'\ B$ unloaded, the algebraic sum of the moments to the left of the left edge of the turntable T, is less than the sum of the moments due to the dead weight of the bridge and turntable to the right of T, *i. e.*, if, when thus loaded, the points D' and B do not move from their supports. In the latter case, the chords between D and D' would not remain horizontal, thus destroying the condition upon which we have determined the constants of integration.

The expression $\frac{n^2\,w'}{2\,(N-1)}$ is, of course, inapplicable to determining the maximum shearing strains due to partial loads in simple trusses of this class, for the reason that the point of contrary flexure, and the consequent value of N changes with every change in the position of the load.

These changes are too remotely related to admit of simple analytic expression.

The usual assumption that the panel points are successively fully loaded before any part of the moving load goes forward to the panel in

advance is, however, less inaccurate in this case, than in that of simple trusses supported at both ends, because the point of contrary flexure recedes as the load advances.

On this assumption it is only necessary to determine from the equation of the elastic curve the reactions at A, with the panel points successively loaded.

Thus the reaction at A, for any panel load w' at n, (measured from the fixed end D), is $R_n = w' n^2 \frac{(3N - n.)}{2N^3}$ and with the head of the moving load covering the panel point n, from D the whole load is $n w'$, and the whole reaction from it at A, is the sum of the reactions determined by the above equation for the several panel loads 1 to n, making n equal to each of the natural numbers 1 to n.

The difference between the whole load $n w'$ and the sum of these reactions, is the shear due to the partial load on the brace transmitting it from the point n towards A.

In order that a drawbridge, in which the points of support are in a horizontal plane, may be freely moved to and from its abutment supports, it must be constructed with reversed camber equal to the amount of deflection due to the dead load when the ends are unsupported, so that when open, the dead load makes the lower chord horizontal.

It follows from this condition that the bridge in use is subjected to the combined effect of the strains from the moving load on the assumptions above discussed, and from the dead load supported at the center pier.

The latter strains, as well as the dead load deflection, to be provided for in construction, are determined by the well-known rules for semibeams.

The algebraic sums of the strains thus determined, and those of the corresponding members due to the moving load computed by the formulæ above discussed, give the total strains on this structure when in use.

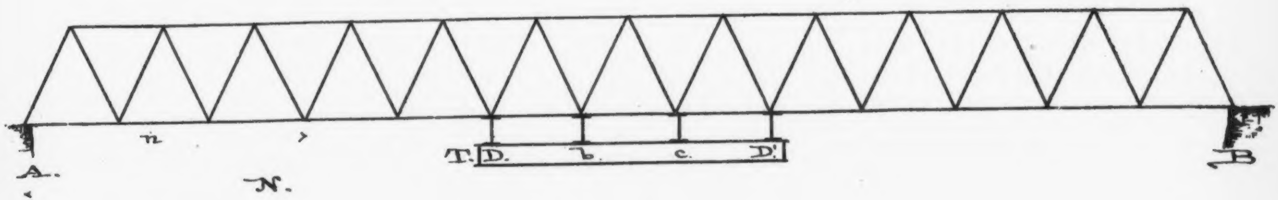


PLATE XXVI
TRANS. AM. SOC. CIV. ENG'R'S.
VOL. IX No. CCX
SWEET ON STRAINS

Fig 3.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

CCXI.

(Vol. IX.—November, 1880.)

DISCUSSION

UPON INTER-OCEANIC CANAL PROJECTS.*

ALSO

ADDITIONAL INFORMATION OBTAINED BY RECENT SURVEYS IN NICARAGUA.

By A. G. MENOCAL, Member of the Society.

I have read with much interest the valuable papers contributed by Messrs. W. W. Evans, J. C. Campbell, Max E. Schmidt, Ashbel Welch, and others, published in the Transactions of the Society, and bearing on the pending discussion on Inter-Oceanic Canal communication.

* Inter-Oceanic Canal Projects, No. CLXXXVIII., Vol. VIII., page 311 (November, 1879). Also Discussions, Vol. IX. (January, February, March and April, 1880).

It seems proper for me to reply to certain statements made and objections raised by those gentlemen as to the merits of the proposed canal across Nicaragua, advocated by me in my paper read before the Society November 9, 1879, inasmuch as some of the objections urged are based on a misunderstanding of the facts, and on other points a need of additional information is shown in order to ensure a full appreciation by all.

I will endeavor to be brief in my remarks, confining them to the elucidation of points bearing on the practicability of that route, and its approximate cost, disregarding some questions of detail in dimensions and methods of construction, which, however much they may be modified, within reasonable limits, to meet the views of different parties, will not materially change the estimated cost of the work.

The paper on "A Project for a Canal Across Nicaragua," now being discussed by the Society, was prepared with the object of showing the advantages of that route in comparison with others, and the approximate cost of the work—some of its important dimensions having been fixed by official instructions. It was based on data obtained by a careful examination of the ground, and an actual location of the line, but neither the dimensions adopted, nor the exact location, were intended to be final. On the contrary, it has been repeatedly stated that the details of construction and location, as projected, will have to be modified to meet the views of those under whose direction the canal may be undertaken and to secure all the advantages which an exhaustive survey will certainly suggest. Explorations of the region traversed by the canal line made subsequent to the official survey, have demonstrated beyond question the certainty of an improvement as to location.

I agree with the distinguished engineers above named, and others, that the locks should not be less than 500 feet in length, but on the other hand, they must admit that their lift may also be increased from 10 to 15 or 20 feet, and the number of such structures reduced thereby, so that the additional cost necessary by the first alteration will be compensated in part, at least, by the second and the probable advantage of better location. The same may be said of the proposed dams across the river San Juan. Their number can be reduced to two, and the new fall of 20 and 34 feet resulting from the change, cannot alarm any engineer when sure of solid homogeneous foundations and good materials of construction. Such a change will not only reduce the cost of the work considerably, but, what is greatly desirable, increase the depth and width

of the river channel, and so facilitate navigation. When these and other modifications, and the necessary changes in the location are introduced in the original plans and estimates, it will be seen that the saving effected thereby will equal or exceed the cost of certain alterations that may be found necessary. I am now engaged in making a verification of certain portions of the first survey, by which a reduction of the estimate of no less than $2,000 000 is confidently expected. The discussion of the detailed dimensions of the several parts of the work proposed seem to me premature, and a waste of time. We are now considering the problem in its general conditions and such essential features as may effect its practicability and probable cost, disregarding those questions which can be settled at any time without affecting any conclusions we may now arrive at.

I will now try to meet the arguments in the order in which they have been presented, and will go out of the Nicaragua route only so far as may be required to sustain certain statements contained in my paper, and bearing on other projects.

Mr. W. W. Evans opens the discussion by several axioms, the accuracy of which cannot be doubted. He might have added, however, that a strait of great width and depth, a Bosphorus or Gibraltar, but free from currents or obstructions of any kind would be the most desirable water communication across the Isthmus. Unfortunately for us, such a strait does not exist, and the question is, can it be artificially made with the means at our command.

I regret very much to disagree with so well known and distinguished an engineer as Mr. Evans, on points touching the problem under consideration, and hope that by throwing more light into the discussion, we may yet arrive at an understanding and agreement as to the essential points of difference between us. Such a result would be extremely gratifying to me.

Mr. Evans objects to the proposed dams across the river San Juan on the sole ground of the danger impending from the failure of such works. The same fears may be entertained as to the stability of retaining walls, bridges, tunnels, &c.; they have also yielded, and will continue to yield, when defectively constructed, or subjected to strains greater than they can resist, but accidents to such constructions have not, as yet, been considered a sufficient argument to prove the necessity of abandoning those systems for overcoming engineering difficulties, nor does there

seem to be any good reason to doubt their stability or utility when properly constructed. The dams proposed across the river San Juan have but a moderate fall, varying from 10 to 23 feet; their sites have been selected where firm foundations and abutments can be obtained, and with those conditions secured, good materials and fair engineering, no fear need be entertained as to their safety. I need not cite examples of existing dams, more than a hundred feet in height, that have been standing for generations, or even centuries, and compared to which the proposed San Juan dams will be lilliputian.

Objections are raised to the location of the canal on the side of a valley and misgivings are felt as to the stability of embankments where heavy rains occur. An examination of the map will readily show that the valley on the north side of the river San Juan, between the San Carlos and San Juanillo, has a very limited water shed drained by but few small streams, and the ground being high, a canal built along that valley, can be effectively drained into the river. From the head of the San Juanillo to Greytown, with the latter river on one side, and the San Juan on the other, a perfect system of drainage can be easily secured for the canal. As to the melting of the embankments during the rain, the objection may to some extent prove to be real in the valley of the San Juan, at certain periods when the rain fall is greatest; but the objection, when applied to the region between the lake and the Pacific, is entirely groundless. There no rain falls for five months in the year, and the total precipitation does not exceed 55 inches. With the rapid growing vegetation found along the whole line of the canal, and the firm soil composed as it is of clay and gravely clay, this danger will be found to be less than anticipated.

With all due respect for Mr. Evans' opinion as to the insufficiency of the prices adopted, I would prefer to leave them as originally recommended. Many engineers and contractors of large experience have considered them quite ample, and some have even gone so far as to express their willingness to undertake the works at the prices estimated.

Omissions are likely to occur in the first estimate of a work of this kind, and I admit that some may have been made in the present case; but they are of comparatively little importance, and will not alter materially the figures given. They are, certainly, of no consequence, when compared to those made by Mr. Evans in his estimate for the San Blas route, where no provisions are made for shoring and inside lining of the

tunnel, for shafts for the same, tide locks or basins, or for dredging and other excavations under water in the bays of San Blas and Panama.

Mr. Evans says that the supposition that the rock on the San Blas route is too soft to sustain itself, is "moonshine," and adds that, "we know from mines and cuttings over all that country, and from surface rock that crops out everywhere, just what we may expect." He says that, "we know we have rock in fixed quantities, and, by estimating it as *granite*, we know that the estimate cannot be far wrong." As to the first proposition, I have to confess that such a discovery has escaped my observations during repeated and prolonged visits to the Isthmus ; besides it is well known there are no mines or cuttings in the region of San Blas, except the deep gulleys on the mountain sides, excavated by the scouring action of the waters, and to show the little reliance to be placed on the surface indications, I will state that M. de Lesseps' engineers made a boring near San Pablo, on the line of the Panama Railroad, close to a very large outcropping of rock, and reached a depth of 60 feet through clay and sand without striking the rock. Similar results have been obtained at other places. On the selected site for the proposed dam across the Chagres river, where rock is also seen on the surface, three borings had been commenced when I crossed the Isthmus in the latter part of March last, and although a depth of from 18 to 24 feet had already been reached, no traces of rock had been met with.

Mr. Evans gives us a very interesting description of his journey through the Suez Canal, and, based on his experience of delays and accidents met with in that single trip, and the time required to haul the steamship *Britannic* into her berth at New York, estimates the probable rate of speed of steamers going through the Nicaragua Canal at 2½ miles an hour and 1½ hours to pass each lock. On the San Blas route he estimates, however, 10 hours to go through 30 miles of canal, 8 of which are in tunnel, with no allowance for passing the tide locks or basins, or for time lost in waiting for a favorable state of the tide for going into or leaving the canal in the Bay of Panama.

It seems strange that an engineer, in discussing the time of transit through a canal on the Nicaragua route, should not take the pains to separate the 181½ miles of what he calls "canal" into its component parts, in doing which he would find canalization 62 miles, slack water navigation, admitting nearly ocean speed 63 miles, and lake navigation, admitting ocean speed, 56½ miles. With this explanation "old canal

men and sailors accustomed to handling these huge vessels" will *not* look with a smile on the estimate of 38½ hours to pass through what is called a canal 181½ miles long, having a lockage of 108 feet. It is interesting to note that some engineers of eminence do not agree with Mr. Evans in his estimate of time required for navigation of canals and passage of locks by large ships. General Gillmore's recent report on the Florida Canal (Senate Ex. Doc. No. 154, 46th Congress, 2d Session) has just been printed. The following are data taken from that report: Length of canal, 122 miles; river and approaches, 47 miles; height of summit, 108 feet; number of locks, 15; estimated speed for ship of 3 000 tons in canal, 5 miles per hour; in river, 9, and in Cumberland Sound, 12½ miles; total time of transit, including 15 lockages, 40 hours; excavation, 108 million yards; cost, $50 000 000.

Mr. Evans objects to the prism recommended for the Nicaragua Canal as insufficient, and in the same paper proposes dimensions for the San Blas route, the water section of which is not larger than the one he criticises.

The River San Juan being the outlet of Lake Nicaragua, the water-shed of the latter is, undoubtedly, a portion of that of the river; but with the two lakes, Managua and Nicaragua, with an aggregate area of 2 700 square miles, acting as equalizing reservoirs, a five-inch rain-fall cannot produce any considerable rise in the River San Juan, of which the first 30 miles have only a fall of 24 inches, and may be properly considered as an extension of the lake. With these conditions it is difficult to conceive how a five inch rain in the lake region can develop foot-pounds of energy or force for doing mischief measured by 4 000 millions of tons falling 107 feet, as apprehended by Mr. Evans. It may be proper to state here, and in this connection, that the Government Custom House, situated near the upper part of Castillo Rapids, at an elevation of not more than five feet above the mean level of the river, has been standing for many years, and there is no record of its building site ever having been flooded.

Referring to the doubts which seem to be entertained as to the character and amount of excavation and dredging required in the bottom of the lake and River San Juan, near San Carlos, I will state that, sounding through the mud to a depth of 20 feet or more, both in the lake and river, and extending to some distance from San Carlos, failed to detect any rock. On the west side the excavation, needed to obtain 30 feet depth at the entrance to the canal, will have to be made in gravel and

fissured rock, and will extend some 1 200 feet from the shore of the lake. An examination of the profile will show that the amount of excavation required is not great nor so difficult to compute as suggested by Mr. Evans.

Reference has also been made to high waves and strong winds on the east side of the lake. This statement is quite true for the Chontalis Coast during a brief portion of the year, when the winds blow from the southwest. But in the vicinity of San Carlos, where dredging is proposed, the lake is well protected by the islands to the west, the winds are light, and the waves are never more than 1½ or 2 feet high.

Mr. Evans does not give credence to the reports as to the calms of the Bay of Panama, and probably believes what has been said on that subject also to be "moonshine." Navy men, however, have a different opinion on that point, and are never anxious to be caught in that trap with their sailing vessels.

Mr. John C. Campbell is of the opinion that the surveys made to the present time are insufficient to establish the best location for a ship canal across the Isthmus. That is the first point in his paper in which we disagree. I believe that the several points presenting favorable indications for the construction of a water way have been sufficiently examined to enable us to determine the relative merits of the several routes. The cry for more surveys generally comes from parties not well informed as to what has been accomplished.

It is rather unfortunate that while Mr. Campbell declares as incorrect my statement respecting the yearly rain-fall at Aspinwall and different places in Nicaragua, he is unable to give any figures to sustain his assertion, but confines himself to a repetition of what General Totten has said on that point, and on reports he had heard while at Aspinwall as to the rain-fall at Greytown.

He says that the elevation of 123.75 feet above mean tide, fixed for the summit level of the Panama Lock Canal by the aqueduct at Matachin is unnecessary, and that a good crossing can be had below Gorgona, and another near Barbacoas. This assertion I will regard as untenable unless Mr. Campbell can show by his plans and figures by what methods and at what cost he proposes to overcome the difficulties arising in clearing the nest of high precipitous hills on the north of the Chagres River at and below Matachin, and, what seems to be still more difficult, to retain the surface of the water in the canal at the required elevation across the

low and extensive valleys between the ranges of hills near Gorgona and Barbacoas. At the latter place the railroad approaches the bridge on an embankment of some elevation, and it would be important to know the length and height he would propose for the necessary aqueduct to cross the river at this point.

Mr. Campbell says that in the design for the aqueduct at Matachin I made no allowance for the water way of the arches, increasing thereby the height of the summit level, a remark which I take as implying that such margin was, in his opinion, unnecessary. No engineer familiar with the peculiarities of the River Chagres and its power for doing damage could consider me either too extravagant or timid in that design. Should I have to make another for the same work, and profiting by the teaching of last November's flood, I would feel inclined to enlarge rather than diminish the water way.

I cannot claim that the line between Aspinwall and Panama surveyed by the United States Expedition is the best route for a lock canal connecting those two points. On the contrary, I believe the location of the canal and the feeder routes are capable of important improvement by a more detailed survey. They are, however, lines of actual location, which cannot be said of any other canal route spoken of or estimated upon, across the Isthmus of Panama. Should Mr. Campbell take the trouble to go over the ground upon which the canal and feeder lines have been located, he would understand why seven tunnels and two syphons were found necessary for the feeder, and also the reasons for the estimate of $18 331 343 for culverts and drains on the canal, which he regards as unjustifiable. In crossing low and extensive valleys running between elevated ranges of hills, such as those in the vicinity of Gorgona and Barbacoas, he would find that the amount estimated under that head is not excessive. He would also find that, in that survey the imagination was not so heavily taxed as he presumes, and that although the survey was completed by two field parties in a little less than three months—not two months, as he incorrectly states—the plans presented deserve all the credit that has been claimed for them respecting the amount of information they give as to topography of the line surveyed.

Referring to my statement that there is on the Panama route a total lack of building materials for the construction of the works recommended, he says that, "this is a discovery that no other man has

made," and adds, "that the Panama Railroad has a number of bridges, all with stone piers and abutments, and it is a new idea that walls cannot be built with stone suitable for bridge works." I am indebted to Mr. Campbell for the compliment; but, really, I am not entitled to any credit for what he supposes a new idea. In making my statement I have only followed an old rule established by many authorities and confirmed by practice, that soft and porous conglomerate, such as can be obtained at Buhio Soldado, the best stone quarry between Aspinwall and Panama, is not a suitable material for the construction of canal locks, aqueducts or sea walls exposed to heavy seas. That material has been made to answer in the construction of piers and abutments for bridges, by employing it in heavy masses; but a close observation of those works and of the storehouses at Aspinwall would have shown Mr. Campbell that the stone he recommends is rapidly crumbling to the ground.

With all due respect for the professional experience of Col. Totten, at Panama and elsewhere, I am not disposed to accept as conclusive his estimated cost of a canal with locks at Panama, as given by Mr. Campbell, until I know that his figures are based on data obtained by the location of a line for that purpose, If such a survey has ever been made, I am not aware of it, and it would be extremely interesting to see the plans and profiles, now that we are seeking for information bearing on that important question.

It may be of interest to state here some information recently obtained as to the flow of the river Chagres, the only available source of supply for a canal with locks at Panama. In crossing the Isthmus in the month of March of the present year the volume of that river was so much reduced by the continuous dry weather of the preceding three months, that its channel at some places was not more than 50 feet wide and nine inches deep, with a flow of not more than 200 cubic feet per second. One of the engineers of M. de Lesseps' suite, engaged in the canal survey, told me that the river could be waded at many places with the water below his ankles. How much more its volume was contracted before the rainy season commenced I could not state, but it is evident the supply of water from that river cannot be relied upon for the supply of a lock canal at all times of the year.

Mr. Ashbel Welch's interesting and valuable paper is full of suggestions that deserve the greatest consideration. Many of them, however,

are alluded to in my remarks as to the details of the different parts of the work proposed or have been already referred to in this paper. I will, therefore, confine myself to reply to such objections or propositions only, as I believe to be based on a misapprehension of the natural conditions affecting the solution of the problem.

Mr. Welch objects to the depth of 26 feet, and width of 72 feet at the bottom, proposed for the canal, as insufficient. Vessels drawing more than 24 feet could not navigate it, and those of 42 feet beam could not pass each other. He suggests 28 feet as the proper depth for the canal. This latter depth would, certainly, be preferable, and will probably be ultimately adopted, should the work be undertaken ; but we have before us the experience of the Suez Canal, which with the same dimensions as have been proposed for the one under discussion, has met quite satisfactorily all the demands of navigation. It is not intended that two vessels of the dimensions named by Mr. Welch shall pass at all points. For a traffic of six millions tons a year, it is not likely that more than nine vessels will, on an average, pass through the canal every day ; and with the lake and river comprising about two-thirds of the whole distance across, and with lake Silico seven miles from Greytown, it may well be assumed that the traffic could be so regulated that vessels in transit will not meet each other on any of the narrow reaches. Should the requirements of the traffic demand it, two or more turnouts at convenient places would meet the difficulty.

The slopes of 1½ to 1 for excavations in earth have been accepted as sufficient for the reason that the roots of rapid growing and dense vegetation, reaching to several feet below the water line, in the region of the canal have proved an excellent protection for the banks of the river and will do the same for the canal banks and slopes.

Mr. Welch says that "the dams proposed are sufficient to pass the maximum flow of water from the lake, but not to pass in addition the water that accumulates in the basin of the San Juan, between the lake and the mouth of the San Carlos;" and adds that, "taking thewater-shed of that portion of the river at 1 500 square miles, a rain-fall of one foot may produce a discharge of 400 000 cubic feet per second and a rise of 18 feet over the crest of a dam 1 000 feet long."

Such a rise has never been seen on the river San Juan, not even below its junction with the San Carlos and Serapiqui ; nor is it likely to occur under the conditions named. More than 37 miles of the 63 miles

of the river intended to be utilized for the canal, will be for all practical purposes on a level with the lake, and a rise in that basin would produce a division of the waters, a great portion flowing towards the lake. Moreover, the large channel of the San Juan, greatly increased by a considerable rise would form a reservoir large enough to store the greater portion of the rain-fall named, so that the flow, even under those conditions, cannot attain the magnitude supposed by Mr. Welch. I do not anticipate a rise of more than 6 feet over the crest of the dams, and the height of 10 feet fixed for the abutment wall may be regarded as sufficient.

Mr. Welch proposes to raise the surface of the lake 12 or 15 feet more than I have recommended. However desirable this modification of the plan may be, it could not be carried out without flooding many hundreds of square miles of the most valuable lands of the country, and destroying whole towns and villages situated near the lake, so that the damages done, would not in my judgment, be justified by the advantages gained. The additional rise of from three to four feet proposed by me can be adopted without hesitation, and about two-thirds of the excavation in the lake and river saved thereby.

The following additional information obtained by recent surveys in Nicaragua may be of interest to those members of the Society who have studied the problem of inter-oceanic communication across the American Isthmus.

It has been stated in the official reports of the surveys made by the United States Expeditions of 1872–3, and by other authorities, that two lines had been located for the canal between Lake Nicaragua and the Pacific—one extending from the mouth of the river Lajas, on the lake, to the Port of Brito; the other from the mouth of the river Del Medio to the same terminal point on the Pacific.

The portion where there is a difference in location is between the lake and the first lock on the Pacific Slope, or, in other words, in that portion of the canal where the level of the lake is to be extended westerly. From lock No. 1 they have a location in common. The deepest cuttings required to reach the proposed level of the canal are: For the Lajas line, $43\frac{3}{4}$ feet; for the river Del Medio route, 134 feet; the length of the respective lines being $17\frac{27}{100}$ miles for the former, and $16\frac{33}{100}$ miles for the latter. The estimated cost of the Del Medio route exceeded that of the Lajas by several millions, but after mature considerations, and by reason

of better natural surface drainage, so essential to the stability of a work of this kind built in a country subject to large rain-fall, the river Del Medio route was favored. On this line no water course of considerable size would be taken into the canal, and as its water shed is quite small, no fears were entertained of damages from freshets. On the Lajas route the conditions were dissimilar, the river Grande, a mountain stream of extensive and rapidly inclined water shed and precipitous channel, approaches the canal from the southwest, and turning to the northwest, passes, with many sinuosities, through a narrow valley of a width but little greater than required for the canal. The channel of the river has an average width of 60 feet, and a depth of from 15 to 20 feet, and in its tortuous course alternately approaches the basis of spurs of a variable elevation, projecting from the main hills on either side. Col. Childs, who, in his plans for a canal by this route, had proposed to receive this stream into the canal by a waste weir, estimated its maximum flow, calculated from the highest water marks on the banks and its mean sectional area and descent, at 5 670 cubic feet per second. I have been able to obtain, as a result of my own observations, and from information furnished by old residents in the vicinity, what may be regarded reliable data as to highest water marks, which, with the mean descent of the river for several miles, and its average water prism, shows that the maximum flow may be as great as 10 000 cubic feet per second. It is quite certain that this stream seldom rises to the height assumed, and since its water shed is very abrupt it remains but a few hours at the high water marks pointed out to me. But so large a volume of water could not be received into the canal, at a time when least needed as a feeder, even under the most favorable conditions of flow, without danger to navigation and to the stability of the works. This was the greatest objection to this route, and to obviate it, the preference was given, not without reluctance, to the river Del Medio route.

Partial examinations of this region, conducted during visits to Nicaragua, subsequent to the official surveys, had greatly added to my former knowledge of the territory traversed by the canal. I was thus aware that important changes in the original location could be introduced, but there was required an instrumental examination in order to determine the measure of the benefits to be derived therefrom. My last survey on this portion of the canal route, made under orders from the Government, was for the special purpose of ascertaining the practicability of

turning the river Grande into the lake, thereby leaving the narrow valley now occupied by its channel across the divide, free for the construction of the canal. With this purpose in view, the survey was begun at a well-known station and bench mark of the survey of 1872, and carried along the bed of the river Grande to its confluence with the river Cascabel, its main tributary, Plate XXVII. Special care was taken in noting all desirable sites for dams. Ascending the stream, the first was found at "El Carmen," a point 3 000 feet, by the river, from the line of the canal. At this point the bed of the stream is $108\frac{48}{100}$ feet, and the highest water mark $124\frac{30}{100}$ feet above mean sea level. From the site of the proposed dam a transit and level line was run in a southwesterly direction, crossing the divide between the Grande and Lajas with an elevation of $178\frac{38}{100}$ feet (see profile of artificial channel, Plate XXVIII). The line then fell into the brook "Comalcagun," an affluent of the river Juan Davila, the main branch of the Lajas. Following the brook across the extensive valley of Ivcotes, the Juan Davila was reached at a distance of $3\frac{88}{100}$ miles from the bank of the river Grande, and at an elevation of $128\frac{27}{100}$ feet.

To turn the river Grande into the Juan Davila, and through the latter and the Lajas, into the lake, a dam $39\frac{67}{100}$ feet high will be required, this height being the aggregate of the difference of level between the river Grande and Juan Davila, and the necessary fall of the artificial channel, as follows:

Elevation of bottom of river Grande............	110.87	
" high water mark of river Grande....	127.37	
		16.50 ft.
" water in Juan Davila..............	128.27	0.90 "
" high water in Juan Davila..........	140.73	
		12.46 "
Fall of artificial channel at 2.53 feet per mile.....		9.81 "
Height of dam above bottom of river Grande =		39.67 ft.

The required height of the dam being established, it was observed that in times of high floods the water in the artificial channel would inundate to the depth of two or three feet the valley of Jobito, and possibly escape into the river through some pass between the hills. Another site for a dam was, in consequence, fixed some distance above the first, and the line for the artificial channel was relocated over higher ground.

The profile, Plate XXVIII., represents the corrected line, and elevation of dam resulting therefrom. The proposed channel is 75 feet wide at the bottom, 15 feet deep, has slopes of 1 to 1, and 2 to 1 respectively, for rock and earth excavation, and with the proposed fall of 2.53 feet per mile will have a discharging capacity of 10 273 cubic feet per second, or more than the largest estimated flood of the river Grande. The elevation of the water of the Juan Davila at its confluence with the artificial channel is $128\frac{27}{100}$ feet in the dry season, and $140\frac{73}{100}$ feet at high floods, or 18 feet and 30 feet respectively above high lake; and the distance from the river being about six miles, there will be a fall of from three to six feet per mile, which in the broad channels of the Juan Davila and Lajas, need occasion no apprehension of the waters rising upwards of nine feet above highest recorded floods, backing into the Grande and flowing over the dam.

Having established the practicability of diverting the river Grande, I directed my attention to a careful examination of the narrow valley occupied by the river, having in view a relocation of the canal route so as to utilize the channel of the river, wherever practicable, avoid the hills on both sides, and enlarge the radii of the curves as much as the topography of the country would permit. The relocation made, as shown by the map and profile, Plates XXVII. and XXVIII., is more favorable than was expected, and the estimated cost of the work, as given below, will show the large amount saved by the substitution of this route for that of the river Del Medio, while the drainage of the latter is, in no respects, better than that of the former.

In computing the amount of excavation required by the new profile, a uniform width of 72 feet at the bottom, and a depth of 26 feet, with slopes as before, have been assumed throughout. That being the maximum width adopted for the river Del Medio route, a fair comparison of the relative cost of the two lines can be made, leaving for another place the consideration of the necessary increase of cost resulting from a modification of the dimensions originally recommended; changes that would affect one as well as the other route.

The following is the amount of excavation and embankment required by both routes, with their estimated cost :

PLATE XXVII.
TRANS. AM. SOC. CIV. ENG'R'S.
VOL. IX. No. CCXI.
MENOCAL.
ON INTER-OCEANIC CANAL PROJECTS.

U.S. NICARAGUA SURVEY, 1872.

WESTERN DIVISION.

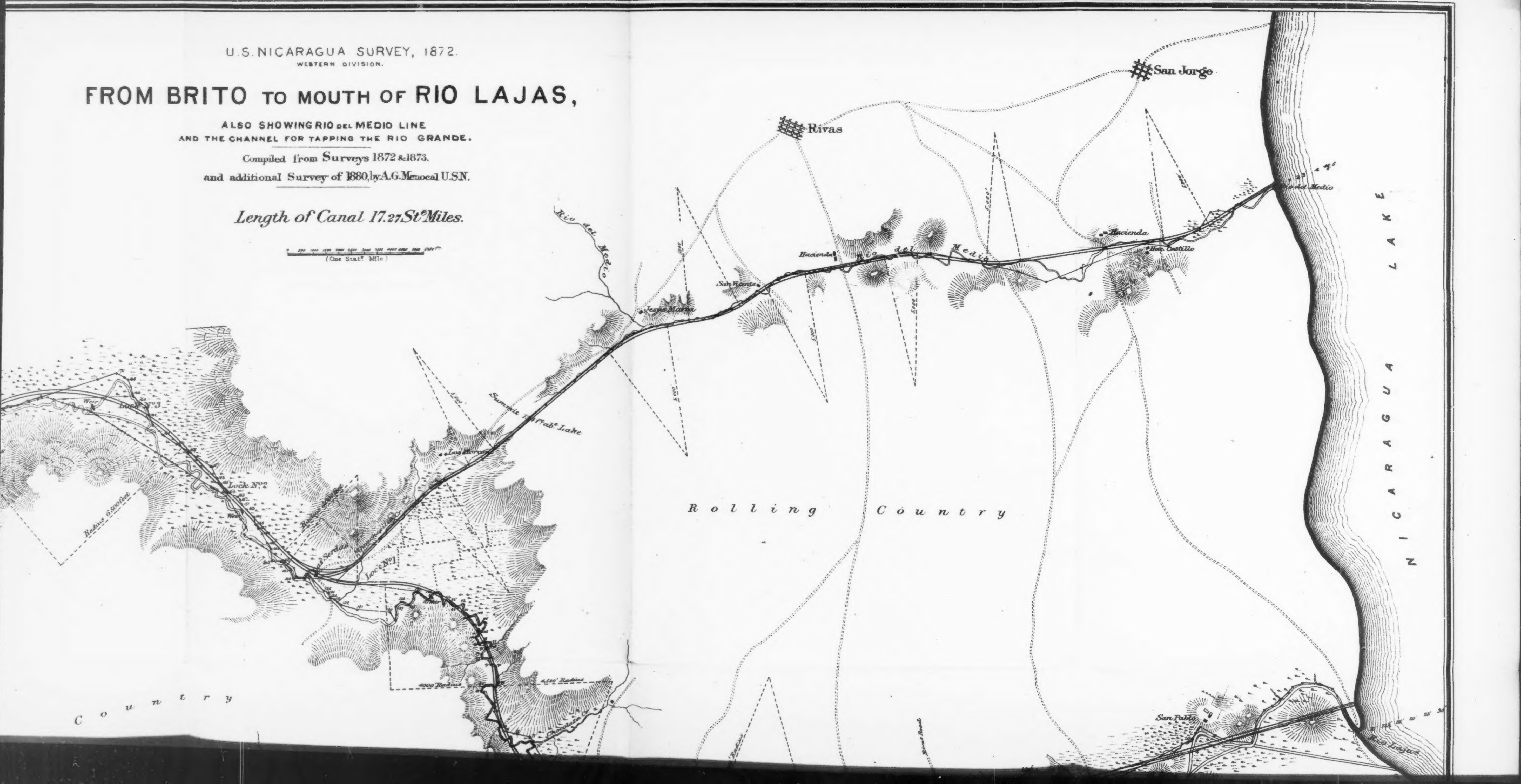
FROM BRITO TO MOUTH OF RIO LAJAS,

ALSO SHOWING RIO DEL MEDIO LINE
AND THE CHANNEL FOR TAPPING THE RIO GRANDE.

Compiled from Surveys 1872 & 1873.
and additional Survey of 1880, by A. G. Menocal U.S.N.

Length of Canal 17.27 St. Miles.

(One Stat. Mile)


U.S. NICARAGUA SURVEY, 1872.
WESTERN DIVISION.
FROM BRITO TO MOUTH OF RIO LAJAS,
ALSO SHOWING RIO DEL MEDIO LINE
AND THE CHANNEL FOR TAPPING THE RIO GRANDE.
Compiled from Surveys 1872 & 1873.
and additional Survey of 1880, by A. G. Menocal U.S.N.
Length of Canal 17.27 St. Miles.
(One Stat. Mile)
San Jorge
Rivas
Rio del Medio
Hacienda
Hac. Castillo
San Pablo
Rio Lajas
NICARAGUA LAKE
Summit 110 ft. ab. Lake
Lock No. 3
Lock No. 2
Lock No. 1
Radius 6,500 feet
4000' Radius
4,500' Radius
Rolling Country
Country

PACIFIC OCEAN
Rough Country
Tide Lock
Lock No 6
Rio Grande
Weir
Artificial Channel for tapping Rio Grande
Comalcagua Cr.
Juan Davila
Rio Juan Davila
Recreo
Jobito
Chocolate Cr.
4000' Radius

Country
4000' Radius
4500' Radius
San Pablo
La Palma
Rio Lajas
Rio Grande
Artificial Channel for tapping Rio Grande
Comalcagua Cr.
Rio Juan Davila
Juan Davila
Rio Lajas
James B. Philp, Draughtsman.

	Cubic Yards.		Price.	Estimated Cost.	
	Del Medio.	Lajas.		Del Medio.	Lajas.
Excavation in rock................	2 915 812	1 359 582	$1 50	$4 373 718	$2 039 373
Excavation in rock above prism....	7 907 812	2 719 165	1 25	9 884 765	3 398 956
Excavation in earth................	6 583 788	7 224 282	0 35	2 304 326	2 528 498
Embankment..................	2 247 565	781 051	0 10	224 757	78 105
Total................	....	....	....	$16 787 566	$8 044 932
				8 044 932	
Difference..............	....	....	....	$8 742 634	

For a fair comparison of the two routes, the cost of the dam and artificial channels should be added to the estimated cost of the Lajas line.

Estimated cost of cutting the artificial channel:

1 592 434 cubic yards excavation in earth, at 35c............	$557 352
403 649 cubic yards excavation in rock, at $1.25............	504 561
Total......................................	$1 061 913
Estimated cost of dam across river Grande...............	140 000
Artificial channel at the mouth of river Lajas.............	488 000
Total......................................	$1 689 913

Deducting this amount from the difference in the estimated cost above given, we have the total saving (under the same conditions and price for quantity of work) of $7 052 721.

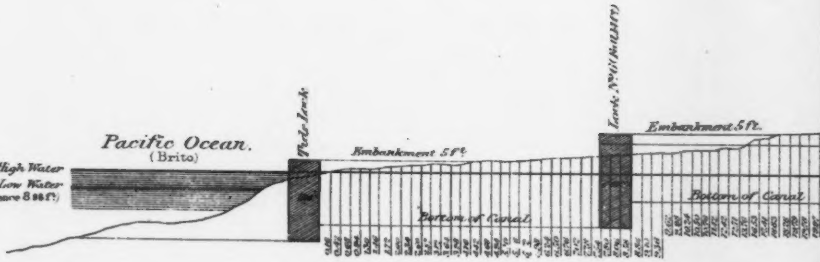
It will be observed from an examination of the profile submitted, that the level of the lake has been assumed at $110\frac{63}{100}$ feet above mean tide, the height it attained at the end of the rainy season of 1878, and that the different reaches of the canal below the first lock have been located almost altogether in excavation, which is not the case on the Del Medio route; so that there has been an increase of excavation and a proportioned decrease of embankment in that portion of the line. The figures in the estimate for the Del Medio route have not, however, been changed.

The height assumed for the summit level, $110\frac{63}{100}$ feet, an increase of three feet to that proposed in the original plans, will bring a decrease of

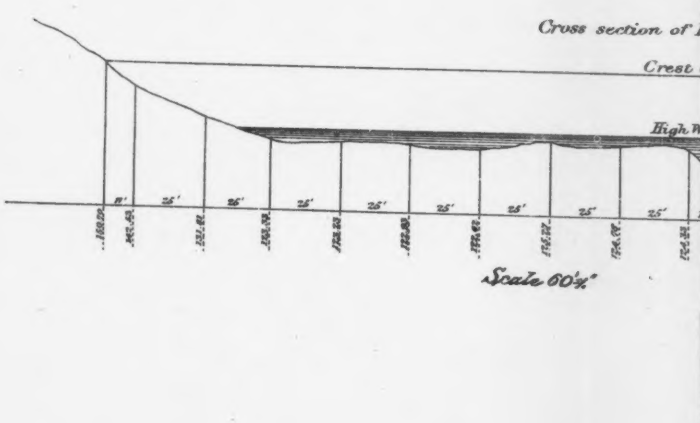
2 380 980 cubic yards of dredging and blasting under water in the lake and river, which, at the prices adopted, amount to $1 356 900.

In the original plan for slack-water navigation in the river San Juan, it was recommended to build four dams, varying in height from 21 to 32 feet above the bottom of the river, and three short canals with locks to connect the different reaches. An excellent site has been found above the river San Carlos, where a single dam of over 2 000 feet in length and 59 feet in height above the bottom of the river, or 49 feet above the surface of the water, will secure an uninterrupted river navigation a distance of more than 63 miles, or a continuous summit level of more than 130 miles between the first locks on either side. The advantages to be gained by this modification cannot be over estimated, not only on account of the reduced cost, but, what is of very great importance, the large increase in depth and width of channel gained in the whole river navigation. The estimated cost of the four dams proposed in the original plans amounts, in the aggregate, to $1 543 526, and that of the short canals to $1 056 922. Supposing the single dam now recommended to cost as much as the other four, there will yet result a saving of over $1 000 000, the cost of the small canals, and $283 000 the cost of the designed diversion of the river San Carlos, now uncalled for.

When the canal line extending from below the river San Carlos to Greytown was located by the surveying expedition of 1873, but little information concerning the topography of the country beyond the valley in close proximity to the river could be obtained. The rainy season was near at hand, and the *personnel* of the expedition was too limited to admit of detaching from the main work of location for the purpose of exploring an entirely new region, in which the chances of finding a more favorable location than was known to exist along the left bank of the river were very remote. More than one attempt was made in that direction, but without definite results. The line was, in consequence, carefully surveyed as delineated in the plans accompanying the official reports, and whatever is there shown, may be relied upon as a truthful representation of the natural conditions, and what was proposed has been regarded as entirely feasible. Some time after the completion of that work, I had occasion to make a complete survey of the river San Juanillo and lower valley of the San Juan, a region scarcely known even by the natives of the country. The information then obtained, sustained by subsequent explorations, indicated the possibility of finding a far more advantageous

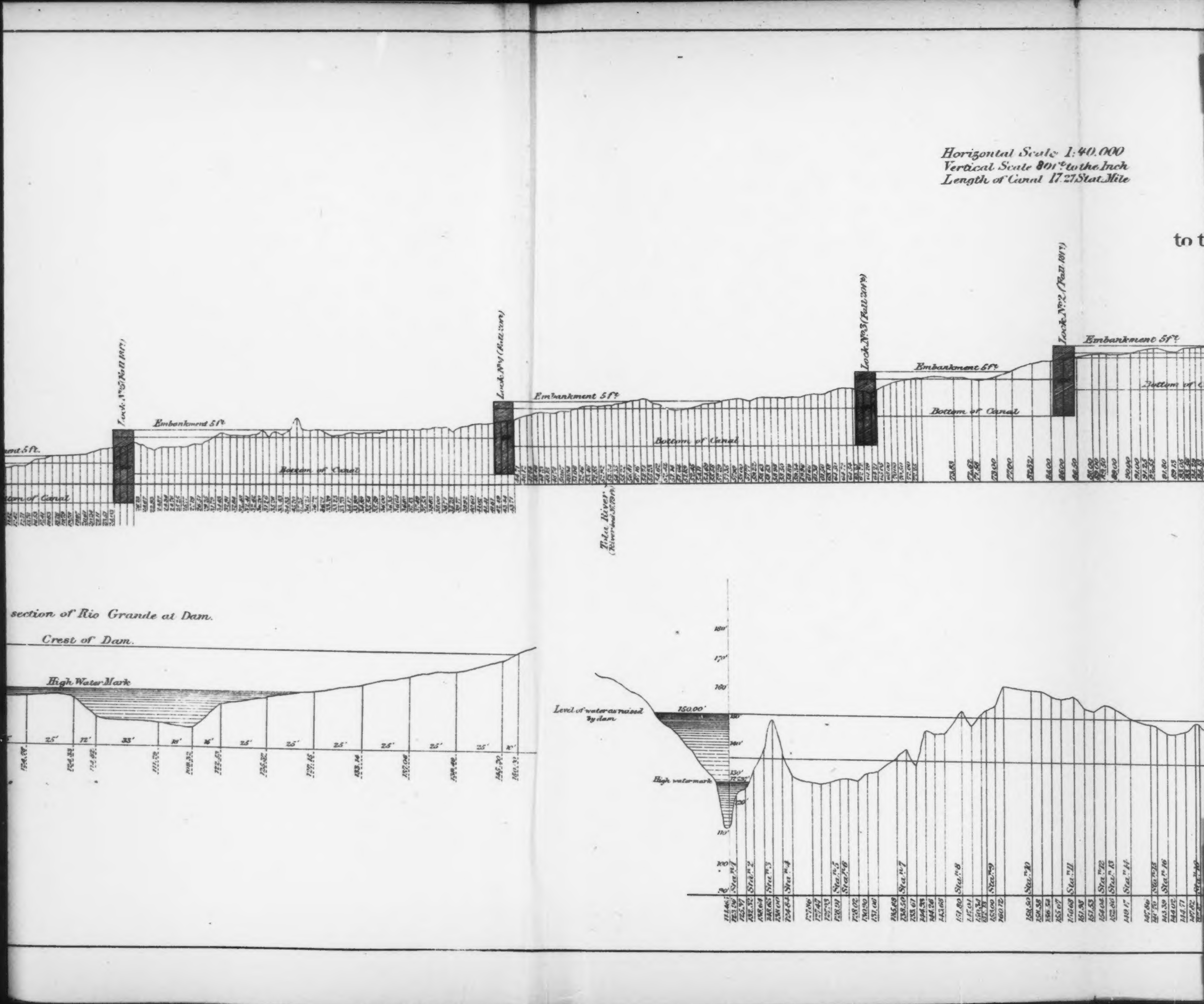


Pacific Ocean.
(Brito)
Mean High Water
Mean Low Water
(Difference 8.98 ft.)
Tide Lock
Embankment 5 ft.
Bottom of Canal
Embankment 5 ft.
Bottom of Canal



Cross section of R
Crest
High W
25'

Scale 60'/1"


Horizontal Scale 1:40.000
Length of Canal 17.27 Stat. Mile
Embankment 5 ft.
Bottom of Canal
Tola River
section of Rio Grande at Dam.
Crest of Dam.
High Water Mark
Level of water as raised by dam
150.00'
High water mark

Profile

OF PROPOSED SHIP CANAL

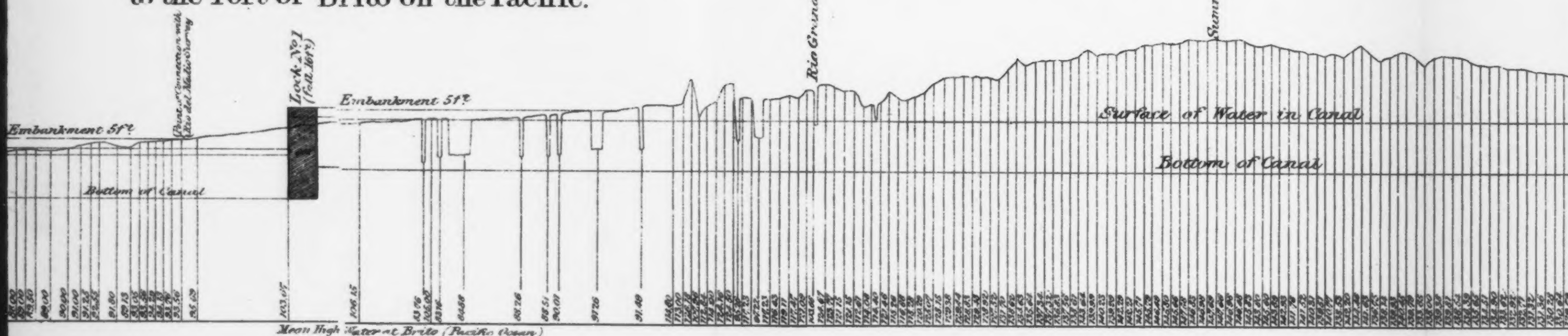
from the Mouth of

RIO LAJAS

On Lake Nicaragua

to the Port of Brito on the Pacific.

40,000
the Inch
Stat. Mile



Profile

OF PROPOSED CHANNEL

for discharging the

RIO GRANDE

into the

RIVER JUAN DAVILA.

Horizontal Scale 800 feet to the Inch
Vertical 40 . . . .
Length of Channel 3.88 St. miles

Depth of Artificial Canal 15 feet

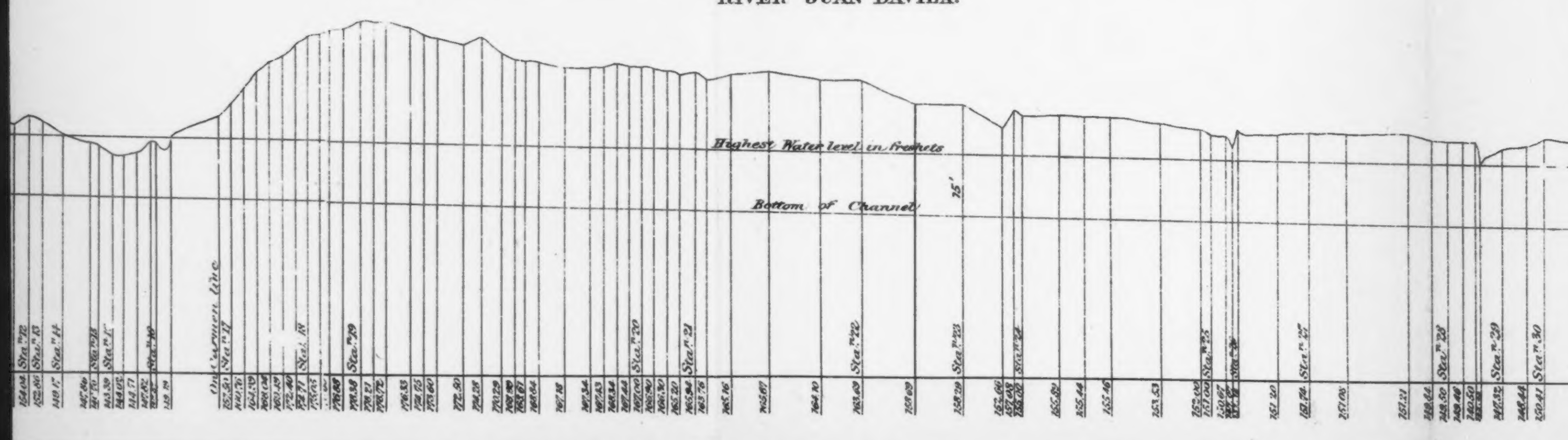
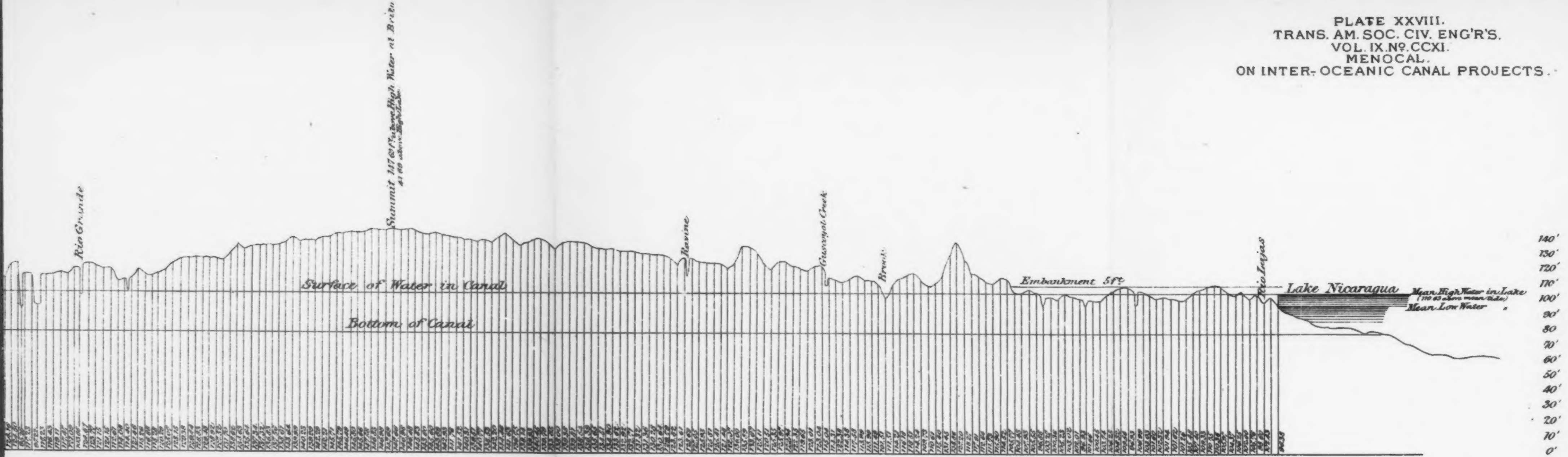


PLATE XXVIII.
TRANS. AM. SOC. CIV. ENG'R'S.
VOL. IX. No. CCXI.
MENOCAL.
ON INTER-OCEANIC CANAL PROJECTS.



Profile
OF PROPOSED CHANNEL
for discharging the
RIO GRANDE
into the
RIVER JUAN DAVILA.

Depth of Artificial Canal 15 feet

James B. Philp, Draughtsman.

location for the canal route, so that diverging the line from the vicinity of the river San Juan at a point seven miles below the mouth of the San Carlos, it would reach the harbor of Greytown by an almost straight line. It was expected by the new location to save several miles of canal, and, what is of greater importance, to dispense with the sharp curves made necessary by the abrupt bends of the river and high spurs projecting from the main hills in the interior to the waters edge, and relieve the canal from the drainage of many square miles of water shed that had to be provided for by the original plan.

I regret very much my inability to present now the results of an instrumental examination, with plans and profiles showing a line of actual location on which to base an estimate of cost, I am able to affirm, however, that the proposed change is entirely practicable. A personal inspection of that region, conducted with the assistance of a pocket compass and aneroid, shows that the canal line can be taken from the left bank of the river San Juan, at a point six miles below the San Carlos dam, and located over very favorable ground, in an almost straight line to Greytown. The distance saved will be about 7.4 miles, as compared with the first location. Deducting from this distance the needed increase in length of canal on account of change of site for dam, we have the total, as the distance saved, six miles.

I can safely state that the new location will be found more favorable than the first; but assuming the cost of excavation and embankment, per mile, to be the same for both lines, we have an economy in construction of $1 917 336.

The total length of the route from the Atlantic to the Pacific with the changes of location herein referred to is:

From the lake to the Pacific	17.27	miles.
Lake navigation	56.50	"
River navigation	63.90	"
Canal from river San Juan to Greytown	35.90	
Total	173.57	miles.

Of which but $53\frac{15}{100}$ miles is actual canal.

The aggregate saving by the several changes proposed, is as follows:

By Lajas route	$7 052 721
Dredging and blasting in lake and river	1 356 900
Short canal in river San Juan	1 056 922
Six miles of canal between the river and Greytown	1 917 336
Total	$11 383 879
Which, deducted from the orginal estimate	52 577 719
We have, estimated cost of the work, on the same basis	$41 193 839

I present these figures in the hope that if interest in the subject continues, such discussion as would add to our information may result.

ERRATA.

Transactions Vol. IX., page 327, August, 1880.

Formula at bottom of page should read—

$$x\,y = -\,bl \text{ (a constant)} \quad (11)$$

Transactions Vol. IX., page 352, September, 1880.

Seventh line, for *best* absorbent, read least absorbent.